

EFFECT OF PAVEMENT TEMPERATURE ON CONCRETE PAVEMENT JOINT RESPONSES

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ABSTRACT

The effect of temperature on joint movement and load transfer efficiency (LTE) was evaluated based on data collected from an instrumented runway at Denver International Airport (DIA). The measured pavement temperature data from DIA was compared with an existing heat transfer model, the Integrated Climatic Model. Good agreement was obtained between the measured and predicted temperature differential and the average pavement temperature for airport concrete pavements. Average pavement temperature significantly affected the dummy and doweled joint movement. Hinged joints were not affected by changes in the average pavement temperature. The doweled and dummy joint movements varied the most in the fall season. The doweled and dummy joint movements were also restricted during certain periods in the summer due to the joint completely closing. The average pavement temperature affected the LTE of dummy joints due to its affect on aggregate interlock. A new correlation was developed between LTE and average pavement temperature for dummy joints. The LTE of dummy joints was found to be less than 50 percent for 55 percent of the year. The LTE of doweled and hinged joints were not significantly affected by changes in average pavement temperature. Hinged joints at DIA were found to be the most effective in transfer load between adjacent slabs.

INTRODUCTION

The two types of temperature variation affecting concrete pavement responses are temperature differential and uniform change in temperature. A temperature difference between the top and bottom of the concrete slab causes curling. The slab weight opposes the curling movement thus inducing stresses in the concrete slab. A uniform temperature change through the slab thickness causes expansion or contraction. Stresses are generated if the slab is restrained from these axial movements. Temperature differential and uniform temperature change can affect the joint opening. As the temperature increases, the concrete slabs expand subsequently narrowing or closing the concrete joints. The joint opening affects the ability of adjacent concrete slabs to transfer load through shear and/or moment. This paper addresses the effect temperature has on concrete joint responses.

Limited airfield pavement studies have focused on the effect of temperature on joint behavior. Kapiri *et al.* [1] found a very good correlation between LTE and average pavement temperature (APT), whereas Foxworthy [2] found a good correlation with air temperature. Brill [3] calculated LTE based on aircraft passes and separated the responses into summer and winter seasons. He found that LTE of dummy and doweled joints were highly dependent on the season. Most studies focused on the effectiveness of different joint type on transferring load including Hammons *et al.* [4,5], Dong and Guo [6], Brill and Guo [7] and Khazanovich and Gotlif [8]. Few studies have also addressed the effect of temperature on joint opening of different joint types. Teller and Sutherland [9] and Kapiri *et al.* [1] were some of these few studies. Most joint opening studies have focused on finding a linear relationship between temperature and joint opening but were not successful because of the variability of the data. Lee and Stoffels [10] reduced the data scattering by introducing a parameter in their analysis called joint-closure temperature.

LTE can either be defined by the deflection or stress transfer across the concrete joint. Since deflections are the easiest to measure, LTE is typically reported in terms of deflection, although LTE can be expressed in terms of stress as seen below:

$$\text{LTE}_{\delta} = \frac{\delta_U}{\delta_L} \times 100 \text{ or } \text{LTE}_{\sigma} = \frac{\sigma_U}{\sigma_L} \times 100 \quad (1)$$

Where,

LTE_{δ} or LTE_{σ} = load transfer efficiency based on deflection or stress

δ_U or σ_U = deflection or stress on the unloaded side

δ_L or σ_L = deflection or stress on the loaded side

Another parameter commonly used to describe load transfer effectiveness is Load Transfer (LT), which is defined in the FAA Advisory Circular 150/5320-6D [11,12] as the total edge stress that is transferred to the unloaded slab. In fact, LT is the parameter used in the FAA design and a value of 25 percent is assumed. LT is expressed mathematically as follows:

$$LT = \frac{\sigma_U}{\sigma_E} \times 100 \quad (2)$$

Where,

$\sigma_E = \sigma_L + \sigma_U$ = stress at the free edge of the slab

Ioannides and Korovesis [13] along with Hammons and Ioannides [14] were among the first to recognize that the relationship between LTE based on stress and deflection depended on the geometry of applied load. Brill [3] studied the effect of aircraft type on LTE based on measured deflection and strain profiles across the joint due to a moving aircraft. Based on Analysis of Variance (ANOVA), he concluded that different aircraft types produce statistically different LTE_{δ} for dummy joints based on deflections measured during the winter months. For the strain-based LTE, Brill found no significant difference was found between aircraft type and measured LTE during either the summer or winter. Ioannides and Korovesis [13] concluded that LTE_{δ} is insensitive to loaded area size whereas LTE_{σ} is sensitive. Rufino *et al.* [15] found that LTE_{δ} depends not only on the load geometry, but also on the relative position between point of the application of the load and point of interest. For instance, the LTE_{δ} based on HWD load at the transverse joint was 25 percent, whereas the LTE_{δ} based on actual aircraft pass was greater. A B-727 traveling across the transverse joint produced a LTE_{δ} of 42 percent at the corner and 28 percent at the transverse joint, whereas a B-777 traveling near the corner produced a LTE_{δ} of about 37 percent at the corner and transverse joint location. Although LTE_{δ} is insensitive to the size of a circular loaded area, it is sensitive to gear configuration or multiple-wheel loads. The effect of gear type on LTE was not evaluated in the present study, which is only based on Heavy Weight Deflectometer (HWD) tests performed on the Federal Aviation Administration (FAA) instrumented runway sections at DIA. The effect of average pavement temperature on LTE and joint movement was evaluated for the following three joint types: dummy, doweled and hinged.

DIA INSTRUMENTATION PROJECT BACKGROUND

Figure 1 shows the location of the joint gages used to measure joint opening, as well as the types of joints within the instrumented section at the DIA. There are a total of ten joint gages: two at the hinged joints (J898 and J905), two at the doweled joint (J897 and J902), and six at dummy joints (J899, J900, J901, J903, J904, and J906). Each working joint gages recorded a total of 12,609 data points. Sensors J901, J903, and J906 were excluded from the analysis due to insufficient data. The location of each HWD testing performed at DIA, as well as the corresponding joint types, is shown in Figure 2. Five periods of HWD testing have been

performed at DIA. Seven different joint locations (T1 to T7) were tested with the load applied to both sides of the joints, whereas two joint locations (TS-1 and TS-2) had only one side loaded.

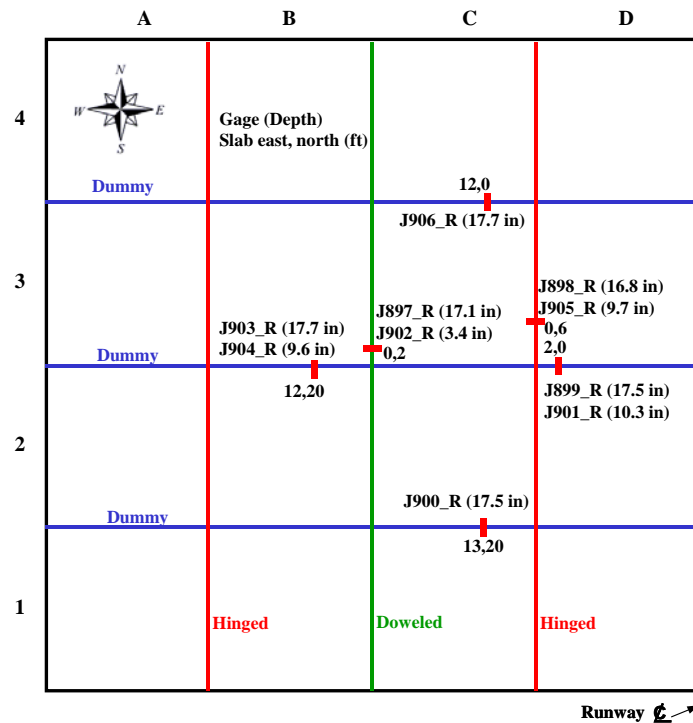


Figure 1. Joint gages and joint types at DIA

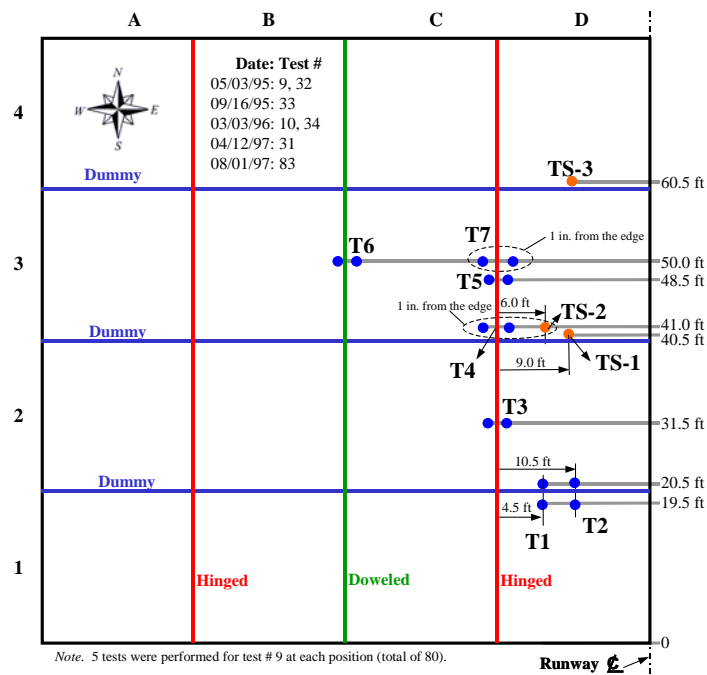


Figure 2. Location of HWD testing

PREDICTION OF PAVEMENT TEMPERATURE PROFILE

Temperature sensors were installed at DIA to collect data on an hourly basis. However, data were not available for all times of the year due to sensor or data acquisition malfunctions. Table 1 presents the number of temperature data recorded by each thermocouple. Considering that almost 8,800 hours exist in one year, measured temperature data is available at most 30 percent of the time within one year.

Table 1. Number of events recorded by thermocouples

Sensor	Slab	1994	1995	1996	1997	1998	1999
T1-T8P2	A2	0	0	0	0	0	0
	B3	0	2016	2200	842	2690	1086
	C3	0	2016	2409	376	2690	1086
T1-T22P10	A3	0	0	0	0	0	0
	B3	0	2016	2200	842	2690	1086
	C3	0	2016	2409	376	2690	1086

As there were many gaps in the measured pavement temperature database, a reliable tool for predicting pavement temperature had to be used before correlating temperature to pavement responses. There are several available approaches to predict pavement temperature with depth given standard climatic data, e.g. see Faraggi *et al.* [16], Barber [17], and Dempsey and Thompson [18]. The last authors developed a heat-transfer model at the University of Illinois, which was later incorporated in the Climatic-Materials-Structural (CMS) program assembled by Dempsey *et al.* [19]. Larson and Dempsey [20] combined the CMS model into a program called Integrated Climatic Model (ICM). ICM 2.6 is the latest version available. ICM is based on one-dimensional coupled heat and moisture flow analysis developed to analyze the interaction between layered systems and climate. ICM is a combination of four different models resulted from a joint effort between Texas Transportation Institute, Texas A & M University and University of Illinois in 1989 (Larson and Dempsey [20]). The model for predicting pavement temperature in the ICM is the CMS model developed at the University of Illinois by Dempsey *et al.* [19]. This heat transfer model considers radiation, convection, and conduction but disregards effects of transpiration, condensation or evaporation. The climatic data required to run ICM are temperature, cloud cover, wind speed and precipitation. ICM accepts daily or hourly climatic values.

The ICM program, more specifically the heat transfer model, has been used by many investigators including application to DIA database (Kapiri *et al.* [1]), development of a mechanistic design procedure for Illinois Department of Transportation (Thompson *et al.* [21]) and the recently developed 2002 Pavement Design Guide (Darter *et al.* [22]). Kapiri *et al.* [1] compared ICM temperature predictions to thermocouple measurements at DIA. Using only maximum and minimum daily temperature data, they concluded that ICM was a valid tool to predict temperature within the pavement structure. Only the predicted temperature near the pavement surface differed slightly from the measured values. In order try to minimize some of the discrepancies reported by Kapiri *et al.* [1], hourly climatic data were used throughout this study instead of daily values.

Hourly weather data at Denver were obtained from the National Climatic Data Center (NCDC). Climatic data from two weather stations were used in this analysis: Stapleton and

Denver International. The reason for using two weather stations was to fill gaps in the data that occurred in the weather station of interest, Denver International. Weather data collection commenced at Denver International on 7/1/1994. Prior to this date, climatic data could only be obtained from Denver Stapleton. When climatic data were not available for specific hours, ICM allowed for interpolation of the data. The present analysis started one year before the period of interest or January 1993 to guarantee stabilization of the predicted values. Between January 1, 1993 and August 31, 2001 there are 75,960 hours. After combining data from the two weather stations, the number of hours with available data was 73,467, which represented approximately 97% of the maximum possible data for the analysis period.

To establish a set of measured temperature values to be compared with predicted temperature value, a program in Visual Basic was written to manipulate the data. Concrete slab temperature data were filtered from both slabs B3 and C3, in order to use only the most reliable data. The final measured temperature file contained 9057 hours of temperature data from 1995 to 1999 based on FAA database. A total of 43,824 hours occurred between these two dates, which meant the sensors were collecting data only about 20 percent of the time.

A program was also written in Visual Basic to create hourly ICM input files. The first step to create ICM files was to extract hourly temperature, wind speed, cloud cover (or percent sunshine) and precipitation from the climatic data obtained data from NCDC. The second step was to combine weather station data (Stapleton and Denver International), giving priority to Denver International. Finally, the ICM input file was created combining the climatic data with date, radiation, and sunrise and sunset times, among other parameters. Table 2 shows concrete material properties used in the sensitivity analysis of ICM.

Table 2. Concrete material properties used in the sensitivity analysis of ICM input values

Parameters	A1	A2	A3	A4	A5	A6	A7	A8
Concrete thermal conductivity (BTU/hr-ft-F)	0.80	0.40	0.60	0.80	0.80	0.80	0.80	0.80
Heat capacity (BTU/lb-F)	0.20	0.20	0.20	0.15	0.25	0.20	0.20	0.20
Emissivity factor	0.65	0.65	0.65	0.65	0.65	0.85	0.93	0.65
Surface short-wave absorbtivity	0.65	0.65	0.65	0.65	0.65	0.65	0.65	0.80

The ability of ICM for predicting pavement temperature was then evaluated by considering all available measured pavement temperatures from DIA database. A program was written to combine the corresponding predicted pavement temperature from ICM output files to the corresponding measured temperature from the FAA database. Figure 3 shows the depth of thermocouples located in both slabs B3 and C3, as well as the ICM nodes. Table 3 shows the analysis of individual temperature prediction. The set of concrete parameters A2 better reproduced the measured average pavement temperature. Table 3 also shows that the average absolute error at the location of the top sensor (T1) and bottom sensor was 3.7°F and 2.2°F, respectively. As the thermocouple depth increased, the predictive error decreased.

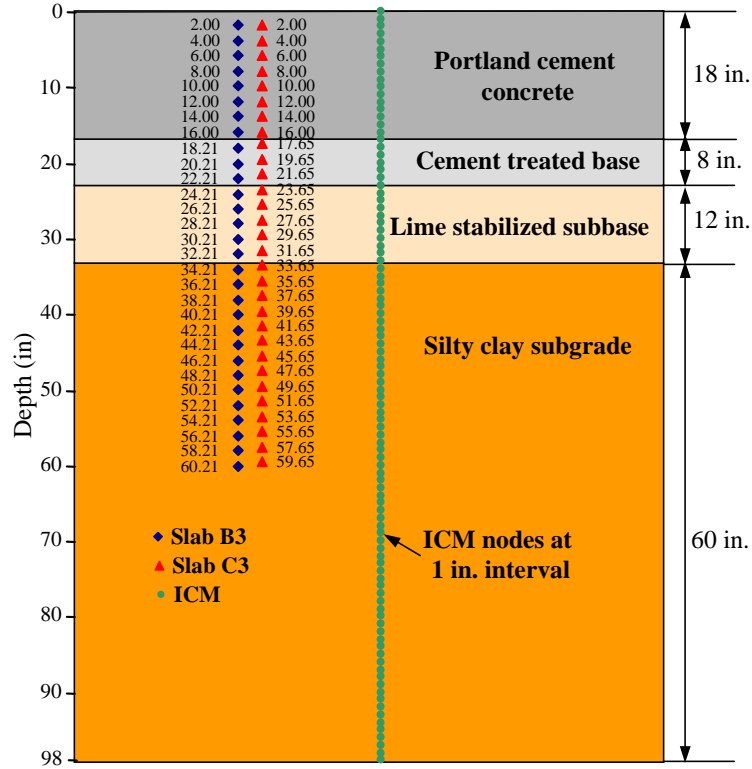


Figure 3. Depth of thermocouples and ICM nodes

Table 3. Absolute average temperature error (°F) between nodal predicted and sensor measured temperature for all available measured temperature data

Sensor	Depth (in)	A1	A2	A3	A4	A5	A6	A7	A8
T1	2.0	4.3	3.7	4.0	5.1	3.7	4.2	4.2	5.0
T2	4.0	3.6	3.5	3.5	4.0	3.5	3.6	3.6	4.0
T3	6.0	2.9	2.8	2.7	3.6	2.7	2.9	2.9	3.4
T4	8.0	2.8	2.7	2.5	3.4	2.6	2.8	2.8	3.2
T5	10.0	2.7	2.5	2.4	3.2	2.5	2.7	2.7	3.1
T6	12.0	2.7	2.4	2.4	3.1	2.5	2.7	2.7	3.0
T7	14.0	2.7	2.3	2.4	3.2	2.5	2.7	2.7	3.0
T8	16.0	2.8	2.2	2.4	3.2	2.6	2.8	2.8	3.1

Table 4 shows the average absolute error between predicted and measured values based on temperature differential and average pavement temperature for each set of concrete properties shown in Table 2. The smallest average absolute errors in predicting temperature differential and average temperature were 3.3°F and 2.4°F, respectively, for the A2 concrete properties. Figure 4 shows the cumulative frequency distribution of measured and predicted temperature differential, whereas Figure 5 shows the same analysis for average pavement temperature. The measured and ICM predicted temperatures are very close in terms of cumulative distribution. This proves ICM does an excellent job predicting pavement temperature based on measured climatic data. For a more detailed evaluation of ICM, see Rufino [23].

Table 4. Absolute average temperature error (°F) for differential and average concrete slab temperature for all available measured temperature data

Analysis	A1	A2	A3	A4	A5	A6	A7	A8
Temperature differential	3.7	3.3	3.5	4.8	3.2	3.7	3.7	4.4
Average temperature	2.8	2.4	2.5	3.3	2.6	2.8	2.8	3.2

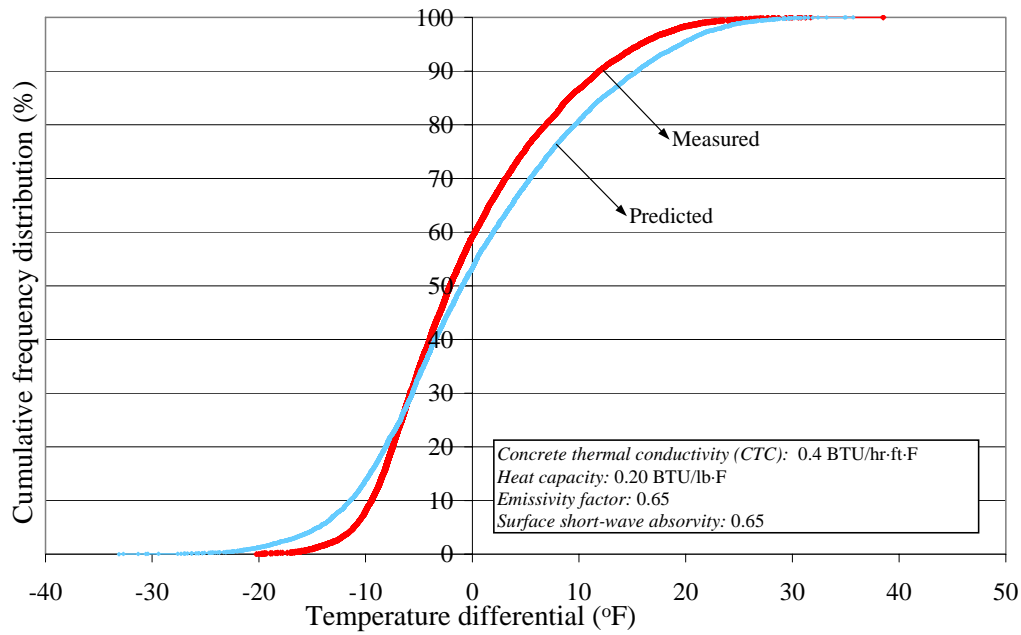


Figure 4. Pavement temperature differential distribution

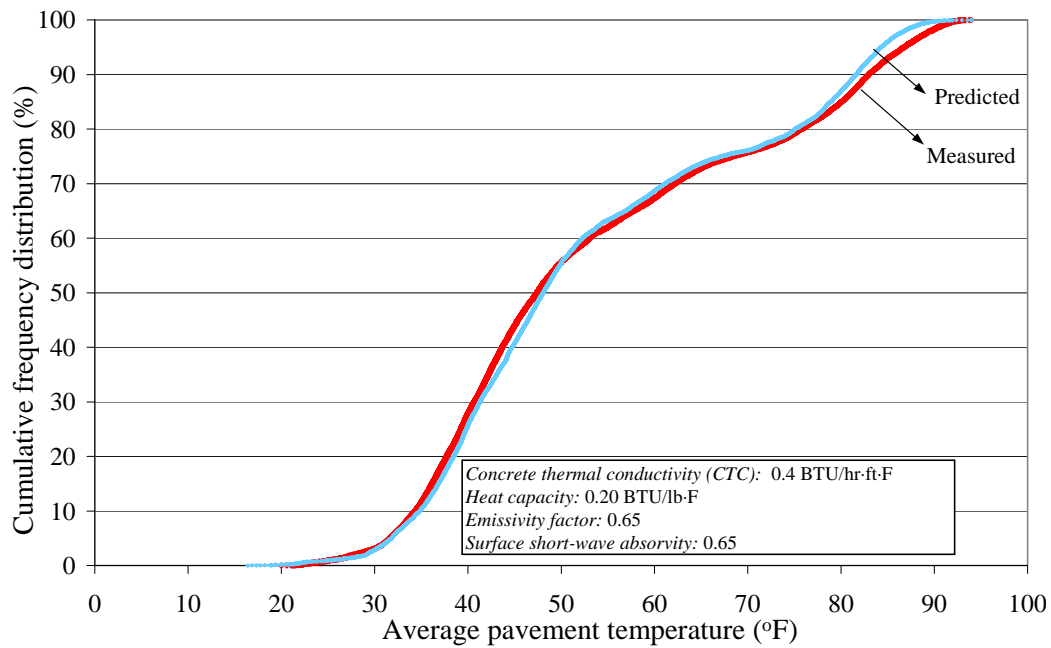


Figure 5. Average pavement temperature distribution

TEMPERATURE EFFECTS ON JOINT MOVEMENT

Joint movement data were first evaluated by relating the joint movement with the predicted temperature data. Only the relative joint movements were measured at DIA, not the actual joint opening magnitude.

Figure 6 shows the relative joint movement and average pavement temperature versus time for the dummy joints. As the relative movement is reduced or becomes more negative, the joint is closing. For the hinged joint, lower joint opening corresponded to lower relative joint movements, whereas higher relative joint movements meant wider joints.

Figure 6 also shows that dummy joint movement was a mirror image of the average pavement temperature. The effect of temperature on doweled joint movements was similar to dummy joints. In contrast, the relative movement of hinged joints was very small.

Figure 7 shows relative hinged joint movement versus pavement average temperature separated by season. The season or average pavement temperature did not affect hinged joint movements resulting in a more consistent joint behavior over time. A summary of joint movement statistics separated by season and joint type is shown in Table 5. The joint movement range and standard deviation in Table 5 show that during the fall, the variation in joint movement was higher than the other seasons. The movement of the hinged joints remained relatively constant throughout the seasons as seen in the low values of standard deviation. The standard deviation of hinged joints was higher during the fall due to the possible outliers, as seen in Figure 7. The comparison between average joint movements, in Table 5, shows that dummy and doweled joints are more closed during the summer time than any other season.

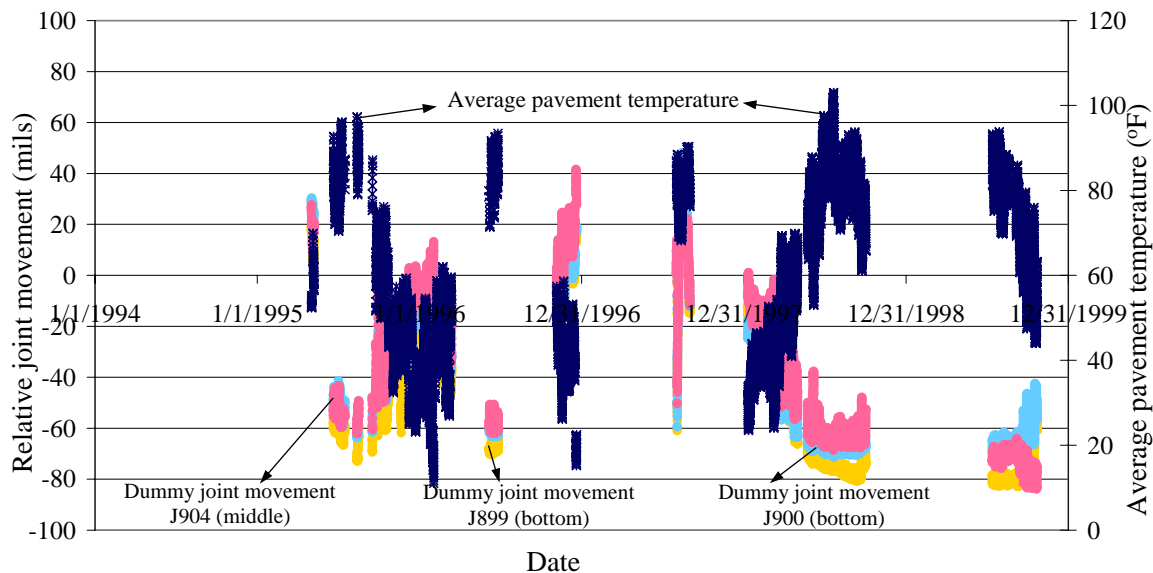


Figure 6. Relative joint movement and average pavement temperature versus time for dummy joints

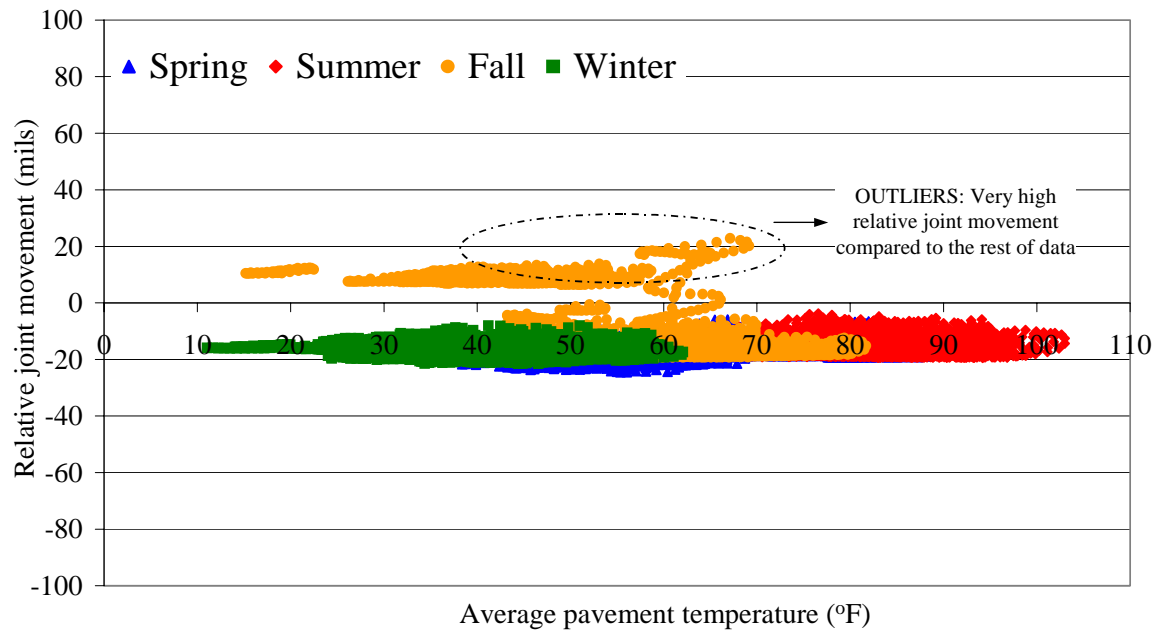


Figure 7. Seasonal joint movement versus average pavement temperature for hinged joints

Figures 8 and 9 show graphically the relative joint movement versus average pavement temperature separated by season for doweled and dummy joints, respectively. Figures 8 and 9 support the findings in Table 5 that doweled and dummy joint movements depend on season. As seen in Figures 8 and 9, the joints open and close more in the fall. There is also a minimum relative joint closing limit, approximately -70 and -80 mils for doweled and dummy joints, respectively. This closing limit primarily occurred in the summer, as seen in Figures 8 and 9, but did happen a small percentage of times in the spring and fall for the dummy joint.

Table 5. Relative joint movement (mils) separated by joint type and season

Statistics	Doweled				Dummy				Hinged			
	Summer	Spring	Fall	Winter	Summer	Spring	Fall	Winter	Summer	Spring	Fall	Winter
Average	-40.8	-35.6	-5.1	-8.7	-69.1	-53.4	-33.3	-21.0	-16.0	-18.0	-8.5	-16.6
Standard deviation	11.4	12.8	24.4	13.6	7.6	12.6	29.4	9.1	3.2	3.4	12.0	2.2
Minimum	-74.2	-72.5	-54.3	-56.7	-82.7	-74.4	-83.7	-45.9	-19.4	-24.5	-19.2	-21.5
Maximum	5.0	-4.4	67.4	25.8	-41.3	-17.6	41.6	13.2	-3.8	-5.9	22.9	-8.0
Range	79.2	68.0	121.7	82.5	41.4	56.8	125.3	59.1	15.7	18.7	42.1	13.5

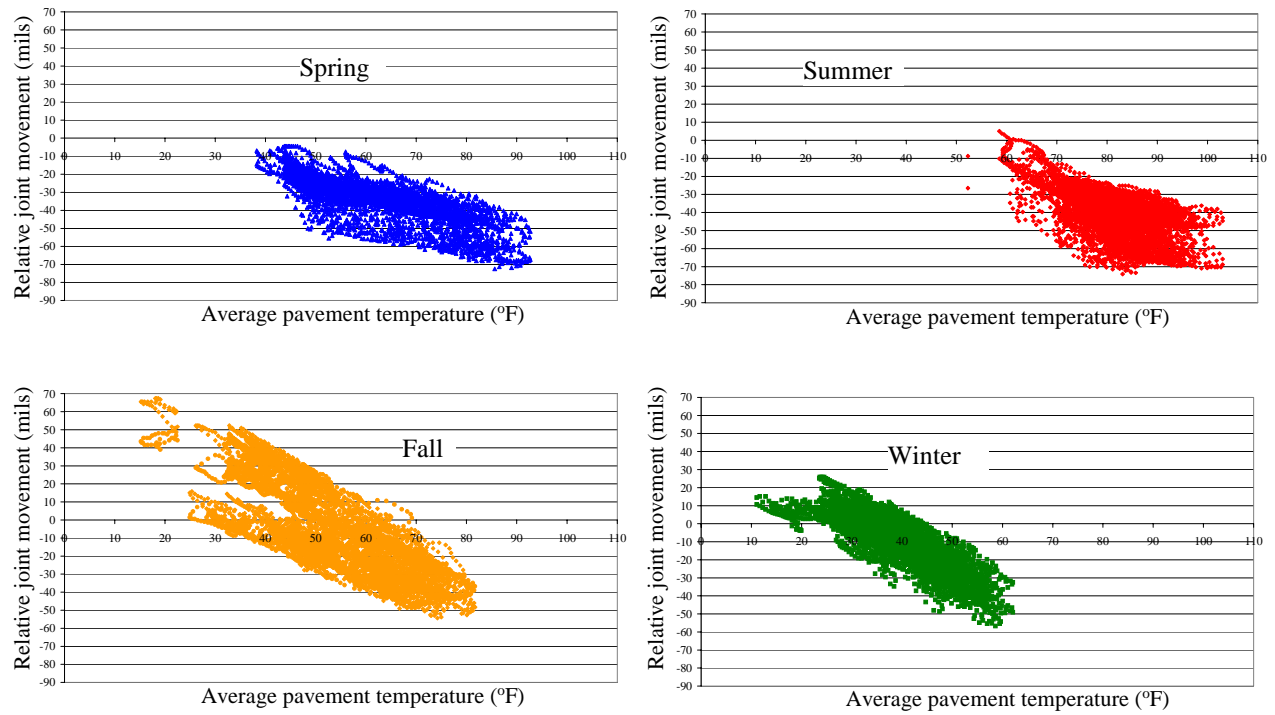


Figure 8. Relative doweled joint movement separated by season

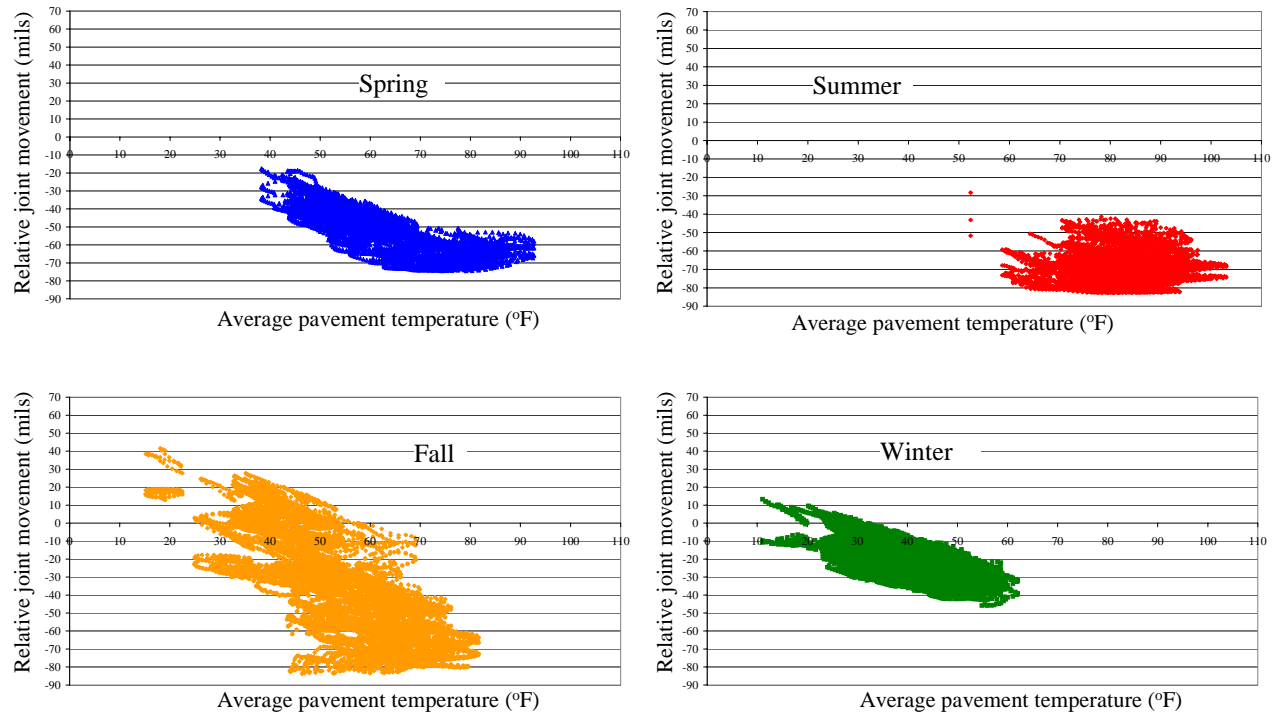


Figure 9. Relative dummy joint movement separated by season

TEMPERATURE EFFECTS ON LOAD TRANSFER EFFICIENCY

Similarly to a previously published study (Kapiri *et al.* [1]), LTE was related to average pavement temperature for several joint types. Figure 10 shows LTE, based on HWD deflection, of doweled joints versus the average pavement temperature. LTE of doweled joints did not change significantly with average pavement temperature. The LTE of doweled joints was approximately 70 percent, which was in good agreement with the Corps of Engineers Ohio Division Laboratories research based on small-scale experiment performed in 1954 (Hammons and Ioannides [14]). The effect of testing direction on doweled LTE was small, as seen in Figure 10.

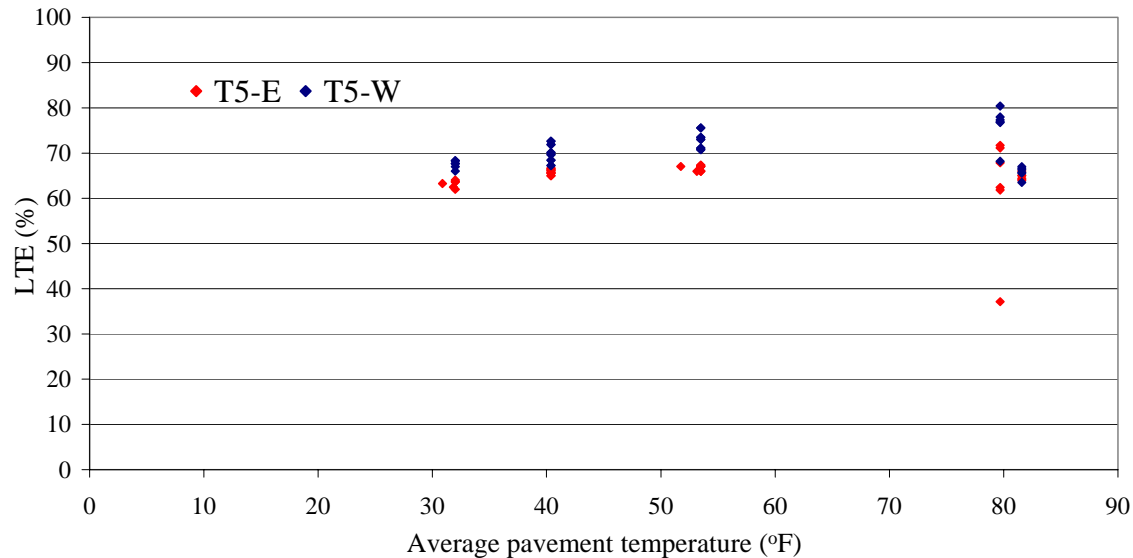


Figure 10. LTE of doweled joints versus predicted average pavement temperature

The LTE of dummy joints was highly dependent upon average pavement temperature, as can be seen from Figure 11. The LTE increased dramatically with average pavement temperature. The direction of testing also did not significantly affect the LTE of dummy joints. Hammons *et al.* [4] tested different Air Force bases located in different climate and concluded that LTE was not affected by test direction, at both low and high load level testing. Figure 11 also shows a regression analysis after excluding outliers. It is observed there is an excellent correlation ($R^2=0.94$) between LTE of dummy joints and average pavement temperature. The LTE of dummy joints at DIA varied from low (10 percent) to high (90 percent), depending on the average pavement temperature. However, it must be emphasized that this correlation may not apply to other dummy joints. Previous research has shown that the shear transfer capabilities of dummy joints is highly dependent on aggregate type and joint opening magnitude (Abdel-Maksoud [24]; and Wattar [25]). This paper also shows that LTE of dummy joints follow the trend of joint movement: as the joint closes in the summer months, the LTE increases.

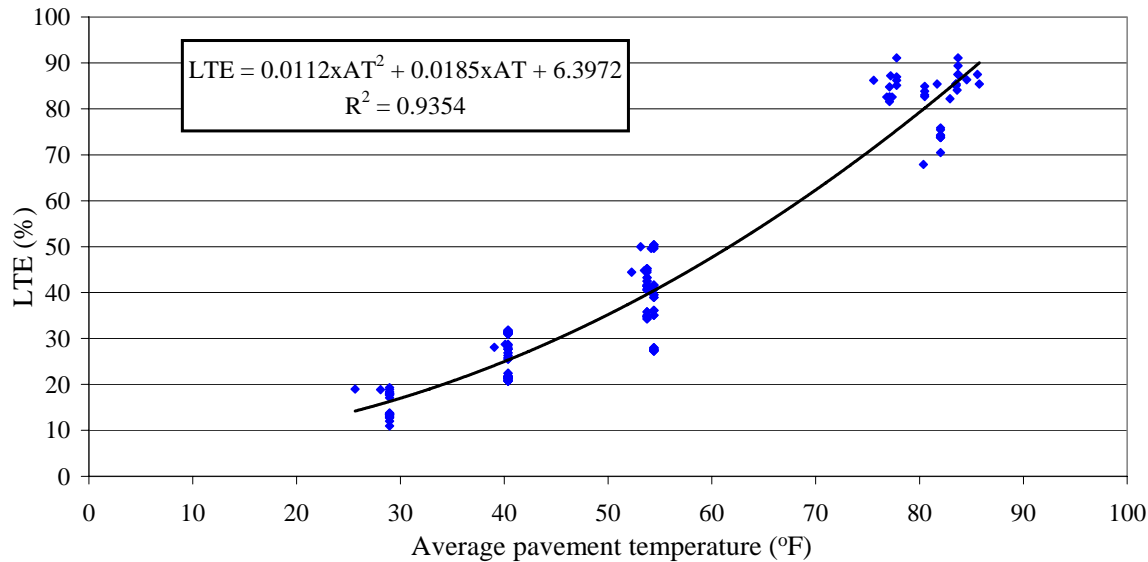


Figure 11. LTE of dummy joints versus predicted average pavement temperature

The correlation between LTE of dummy joints and average pavement temperature can be used to predict the cumulative frequency distribution of LTE for dummy joints. Figure 12 shows predicted LTE for a period of seven years (from 9/1/94 to 8/31/01) based on hourly pavement temperature at DIA. For fifty percent of the time, LTE is below 45 percent. This chart can also be used to evaluate the FAA design guide (FAA, 1995a and 1995b) assumption of load transfer of 25 percent throughout the design period. The LTE based on stresses, from equation 2, is approximately 33 percent. Ioannides *et al.* [26] and Ioannides and Hammons [27] published a relationship between LTE based on stress and deflection from finite element analysis. According to Ioannides and Hammonds [27], a LTE based on stress of 33 percent correspond to a LTE based on deflection of 83 percent. For the DIA database for dummy joints, about 80 percent of the time annually the LTE was below the assumed FAA value of 83 percent. Hammons and Ioannides [27] analyzed data from DIA and concluded that the average predicted LT was close to the assumed design value of 25 percent. However, their analysis was based on doweled and hinged joint data only.

Figure 13 shows LTE did not vary with average pavement temperature for hinged joints. This corresponded with the low measured joint movements for the hinged joints. At DIA, the LTE of hinged joints (75 to 90 percent) was greater than the LTE of doweled joints (60 to 80 percent). Hammons *et al.* [5] and Dong and Guo [6] also concluded that hinged joints provide higher mean LTE than doweled joints. This behavior was already observed in early 1930s when Benkelman [28] concluded that the load transfer through aggregate interlocking is very good if rough cracks are held firmly together.

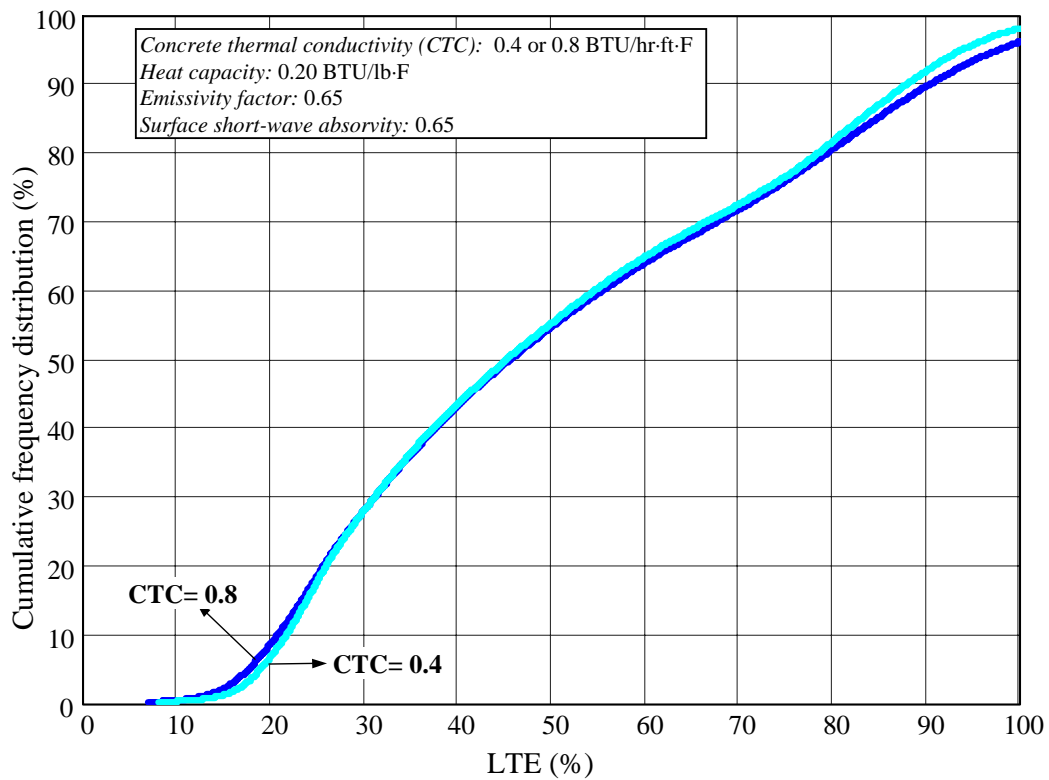


Figure 12. Prediction of LTE of dummy joints based on average pavement temperature for a period of seven years at DIA

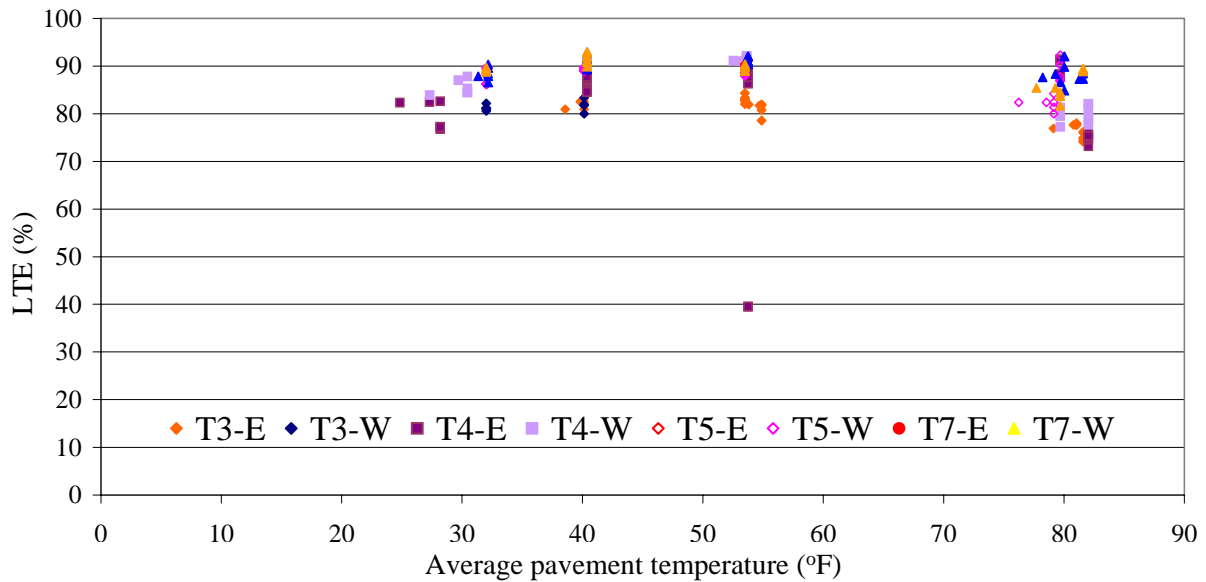


Figure 13. LTE of hinged joints versus average pavement temperature

CONCLUSIONS

Joint movement and pavement temperature data was evaluated from the Denver International Airport database. The measured pavement temperature was predicted using an existing heat transfer model, the Integrated Climatic Model (ICM). ICM was found to predict average pavement temperature and temperature differential distributions with good accuracy.

The effect of average pavement temperature on joint movement was evaluated for three types of joints. Joint movement was a mirror image of the average pavement temperature for dummy and doweled joints. Hinged joints showed less movement than the other two joint types. During summer and spring months, the relative joint movement of doweled and dummy joints was smaller than during winter and fall months. The variation in joint movement was the highest during the fall season.

The effect of temperature on load transfer efficiency (LTE) was evaluated based on Heavy Weight Deflectometer (HWD) deflection readings. Doweled joints had only a slight increase in LTE with increase in average pavement temperature. The LTE of dummy joints was highly dependent upon average pavement temperature. Dummy joints LTE varied from 10 percent to 90 percent depending on the average pavement temperature. A new correlation between LTE and average pavement temperature for dummy joints was developed in this study based on approximately 150 data points. Based on this correlation, the LTE of dummy joints were lower than the value assumed in FAA design guide 80 percent of the time. In contrast, the LTE of hinged joints did not depend on the average pavement temperature. Hinged joints were the most efficient joint type at DIA for transferring load between adjacent slabs.

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